

57-F-248C

C.N.R. OVERPASS

ALLANBURG

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BA 609

Mar 28-4

RACEY, MACCALLUM AND ASSOCIATES  
LIMITED

A COMPANY OWNED, DIRECTED AND OPERATED BY

Consulting Engineers  
AND ASSOCIATED STAFF

MONTREAL  VANCOUVER

TORONTO

DONALD C. MACCALLUM, B.ENG., M.E.I.C., P.ENG.

H. JOHN RACEY, B.SC., M.E.I.C., P.ENG.

A. ERIC RANKINE, B.SC., M.E.I.C., A.M.I.E.L.E.C.E., P.ENG.

TORONTO DIVISION  
27 CARLTON STREET

REPORT NO: S-500-579/T-669

Toronto, Ontario.  
May 10th, 1957.

Department of Highways Ontario,  
c/o J. D. Lee & Co. Ltd.,  
10 Chapin Street,  
KINGSTON, Ontario.

57F-248C

Attention: Mr. H. D. Garbutt.

RE: FOUNDATION INVESTIGATION FOR  
G.N.E. OVERPASS, ALLANBURG,  
ONTARIO.

Dear Sirs:

Attached hereto is our report on the foundation investigation recently carried out at the site of the above noted overpass. For your convenience the results of this investigation are summarized as follows:

1. The sub-soil at the site consists of a strata of prestressed reworked clayey silt of lacustrine origin. This material is in a stiff to very stiff state in-situ.
2. The safe allowable bearing pressure has been determined as 2.4 T/ft.<sup>2</sup>. The settlement associated with this loading intensity has been estimated as 1 1/2".
3. A footing width of 10'6" provides adequate surface to accommodate the lateral forces which would occur from a fill height of 30 feet. The shearing strength of the sub-soil precludes the danger of an embankment foundation failure.
4. The water table elevation has not been accurately established but no water was encountered in the boreholes to a depth of 20 feet. No water problems should be anticipated in excavations

May 10th, 1957.

For the footings.

We are pleased to have been of service to you on this occasion, and we shall be pleased to discuss any matters that may come to mind, after you have read the attached report.

Yours very truly,

RACEY, MACCALLUM AND ASSOCIATES LIMITED.

WAT/AMel.  
Enclosures.

*W. A. Trow*  
W. A. Trow, P. Eng.  
Divisional Soils Engineer.

Department of Highways Ontario,  
c/o J. D. Lee & Co. Ltd.,  
10 Chapman Street,  
KINGSTON, Ontario.

FOUNDATION INVESTIGATION FOR  
G.N.R. OVERPASS, ALLANBURG,  
ONTARIO.

Reference: S-500-679/1-669

Racey, MacCallum and Associates Ltd.  
May 10th, 1957.

May 10th, 1957.

RE: FUNDATION INVESTIGATION FOR  
THE PROPOSED C.N.R. OVERPASS,  
ALLANBURG, ONTARIO.

An investigation to assess the bearing capacity of the sub-soil at the above railway overpass has been completed. A brief description of the field and laboratory work carried out as well as the recommended safe allowable footing loads to the sub-soil are included in the following paragraphs.

LOCATION OF SITE

The proposed structure is located approximately 1/2 mile east of the Welland Canal at the intersection of the C.N.R. line from Welland to Niagara via Allanburg and the relocated portion of Highway No. 20. This intersection is located on lot No. 115, Thorold Township in Welland County.

The crossing site is virtually flat and is surrounded by cultivated fields with natural surface drainage toward the east.

DESCRIPTION OF FIELD AND LABORATORY WORK

The drilling and soil sampling equipment arrived on the site April 10th, 1957 and drilling was started on borehole No. 1 the following day. The locations of the borings and penetration profiles were laid out under the supervision of the soil engineer. A trained soils technician remained on the site and supervised the boring and taking of samples. The field work was completed on April 17th, 1957.

The borings were carried out using a standard diamond drill rig modified for soil sampling. Boreholes numbered 1 and 3 were advanced without the use of wash water, holes numbered 2 and 4 were drilled using the conventional method of alternately driving and washing out the casing. Samples were taken at five foot intervals with this walled 2" Shelby sampler used in the cohesive strata and standard split spoon sampler used in the granular strata. A 2" diameter cone was also driven dynamically at each borehole location, the resistance to penetration of this cone is recorded in blows per foot using an energy of 150 foot lbs. per blow. The cohesive strata encountered were too stiff to carryout in-situ shear tests using a rotating vane.

Immediately upon recovery all undisturbed sample tubes were positively sealed with a low melting point wax to prevent moisture content changes. These samples were kept on site until the borings were completed and then despatched to the Toronto Laboratory, where

May 10th, 1957.

DESCRIPTION OF FIELD AND LABORATORY WORK (Cont'd)

they were visually examined and subjected to such index tests as Unconfined Compression, moisture content, and Atterberg Limits. The results of the field and laboratory work have been summarized on the borehole profiles enclosed in this report.

DISCUSSION OF SOIL TYPES ENCOUNTERED

The results of the four borings carried out at this site show that the sub-soil consists of a fine grained material ranging from a fine silt to a fat clay. The distribution of silt and clay sizes suggests that the original lacustrine deposit was remolded during the period of ice retreat over this area.

The disposition of soil strata as evidenced by the borings is consistent over the area investigated. The surface mantle of dark silty organic top-soil varied in thickness from 1 foot at boreholes numbered 1 and 3 to 1.5 feet at boreholes 2 and 4. Underlying the organic surface layer was a stratum of stiff to very stiff brown clayey silt which contained thin silt seams throughout. The thickness of this stratum varied from 14.5 at hole No. 4 to 21.5 at hole No. 1, with corresponding lower horizon elevations of 567.5 and 566.5. Immediately underlying this stratum a medium strength gray silty clay containing thin seams of red silt was intersected. The thickness of this layer was not positively determined but gravel and shale fragments were noted in the sample recovered from Hole No. 4 at elevation 547.4.

BEARING CAPACITY EVALUATIONS

In order to evaluate the allowable bearing pressure for the abutments to be placed at this site the following characteristics of the upper layer of stiff to very stiff clayey silt have been used.

Natural density =  $\gamma$  = 130 p.c.f.

Undrained shear strength = (determined as  $\frac{1}{2}$  unconfined compression strength)  
=  $c$  = 1000 p.s.f.

In addition footing elevation has been assumed at 580.0 with base of rail elevation at 589.5. This gives a footing depth of approximately 10 feet.

For cohesive materials which exhibit no increase in strength with increased normal load (i.e.  $\phi = 0$  with respect to total stresses) the safe allowable bearing capacity with respect to a shear failure is given by the expression

$$q_s = \frac{cN_c}{F} + p$$

where  $c$  =  $\frac{1}{2}$  unconfined compressive strength



May 10th, 1957.

BEARING CAPACITY EVALUATION (Cont'd)

$H_e$  = bearing capacity factor which is influenced by shape and depth of footing.

$p$  = total overburden pressure at foundation elevation  
 $= \gamma D$

$F$  = Factor of safety = 3

For a continuous footing of width  $B = 11$  feet, and depth  $D = 6$  feet, using values of  $c = 1800$  p.s.f.,  $\gamma = 130$  and  $H_e = 6.2$  (1) the safe allowable bearing capacity given by the above expression

$$q_s = \frac{q_u}{F} + p$$

$$\text{is } q_s = \frac{6.2 \times 1800}{3 \times 2000} + \frac{130 \times 6}{2000} = 2.4 \text{ } \gamma/\text{ft.}^2$$

The above noted safe allowable bearing capacity is obtained considering only the stability of the footing with respect to a complete shear failure. In order to arrive at the settlement that will result from the above bearing pressure Skempton has established an expression which relates settlement to footing width, strength and compressibility characteristics of sub-soil, and footing loads. This relationship is expressed mathematically as follows:-

$$p_s = 5 B \times \frac{c}{E_v} \times \frac{q_u}{q_{uf}}$$

where  $p_s$  = net settlement

$B$  = footing width

$c$  = shear strength

$E_v$  = reciprocal of modulus of compressibility =  $1/\alpha_v$

$\frac{q_u}{q_{uf}}$  = reciprocal of safety factor with respect to net pressures.

Using a  $\alpha_v$  of 150 which is typical for a pre-stressed material as exists  $c$  at this site, a settlement of the order of  $1\frac{1}{2}$  inches is indicated from the above relationship.

In addition to the problem of allowable bearing capacity for the abutment footings the associated problems of horizontal earth pressures against the abutment backwall and overall embankment stability have been considered. The footing width of 10'6" provides adequate resistance to sliding considering a 30 foot fill height even with no passive pressure considered.

1. Skempton A.W. - "Bearing Capacity of Clays", Building Research Congress, 1951.



May 10th, 1957.

BEARING CAPACITY EVALUATIONS (Cont'd)

For embankment heights of 30 to 40 feet the strength of the foundation soil precludes overstressing or possible shear failure.

SUMMARY AND CONCLUSIONS

The results of the foundation investigation at the site of the proposed bridge structure may be summarized as follows:-

1. The sub-soil at the site consists of reworked, prestressed strata of silty clay. The percentages of silt and clay size vary with depth.

No ground water elevation was established due to the low permeability of the sub-soil. Observation during sampling indicate that the water table is about 20 feet below ground surface and no problems due to high water table are anticipated during excavation to footing elevation of 580.0.

2. The safe bearing capacity of the sub-soil has been shown to be  $2.4 \text{ T/ft}^2$ . This value includes the removal of 8 feet of overburden and therefore is an allowable gross pressure at footing elevation.

3. Settlements based on empirical relationships have been estimated at  $1\frac{1}{2}$  for footing loads as given above.



L. G. Soderman. P. Eng.



DEPARTMENT OF HIGHWAYS

Bridge Design Office,  
June 18th, 1959.

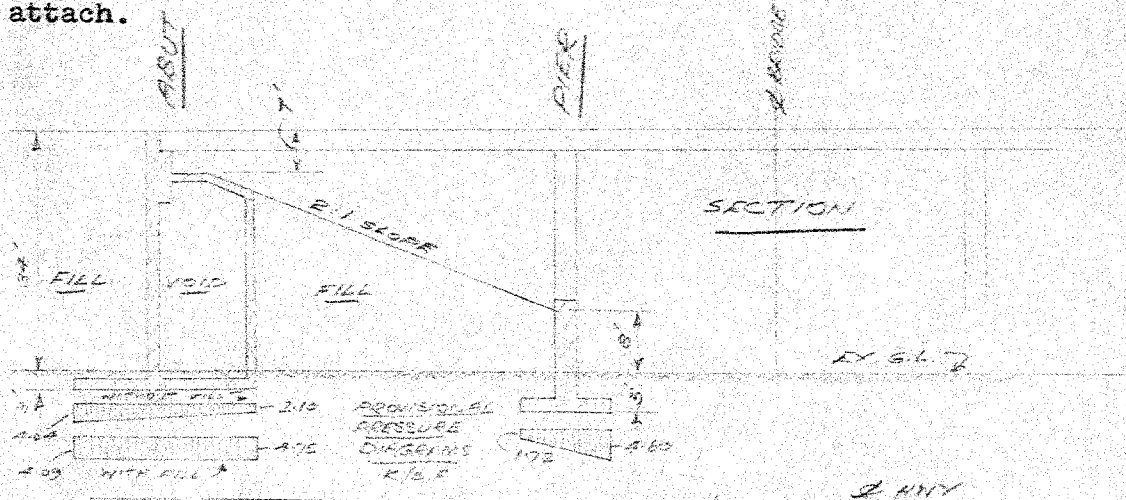
Memorandum to:  
Mr. L. G. Soderman,  
Soils Investigation Branch,  
Downsview

The soils report referred to in our telephone conversation on the 17th June is attached. Please examine the report and suggest permissible bearing pressures for the abutment and the pier foundations sketched below.

If the permissible stress for the abutment is dependent upon the fill, values with and without the fill would be of interest.

*B. S. Richardson*  
B. S. Richardson,  
Bridge Design Office.

BSR/r  
attach.



TWO BOXES TO  
EACH ABUT

PLAN

STRIP FOOTING  
6' LONG

W.P. 537-56

*As this structure has  
been recalled it will  
be filed in spot # 4.*

Mr. A. M. Toye,  
Bridge Engineer.

June 19, 1959.

Materials & Research Section.

Re: Foundation Investigation for  
C.N.R. Overpass, Allanburg, Ontario -  
by Racey, MacCallum & Associates, Ltd.

Attention: Mr. S. McCombie.

In response to a request from Mr. B. S. Richardson of the Bridge Design Office, we have reviewed the proposed footing design for the above noted structure. Our comments, which have been given to Mr. Richardson by telephone, are confirmed as follows:-

1. It has been proposed to found the abutment footings at a depth below existing ground surface, of three feet. In view of the fact that this would result in footings being founded within the upper layer of soil subjected to alternate freezing and thawing, and which now exists in a somewhat loosened state, we feel that the footings should be carried down to at least five feet below existing ground surface. This is consistent with an elevation of 583.0'.
2. The allowable pressure that may be safely transmitted to the subsoil, has been determined as 2 1/2 tons/sq. ft. Increasing the depth of footings, as indicated in Item (1) above, will result in an increase in the footing pressures. Strength characteristics of the subsoil are such that a maximum footing pressure of 3 tons/sq. ft. can be safely tolerated.
3. Differential settlements between the abutment and pier will be of the order of one inch, and this will take place over a considerable number of years. This amount of differential settlement can be readily accommodated by the structure.
4. The general stability of the embankments has been reviewed and there appears to be no danger of embankment foundation failure.

LGS/MdeF

*LCS.*  
L. G. Soderman,  
PRINCIPAL FOUNDATIONS & SOILS ENGINEER.

cc: Mr. B. S. Richardson.

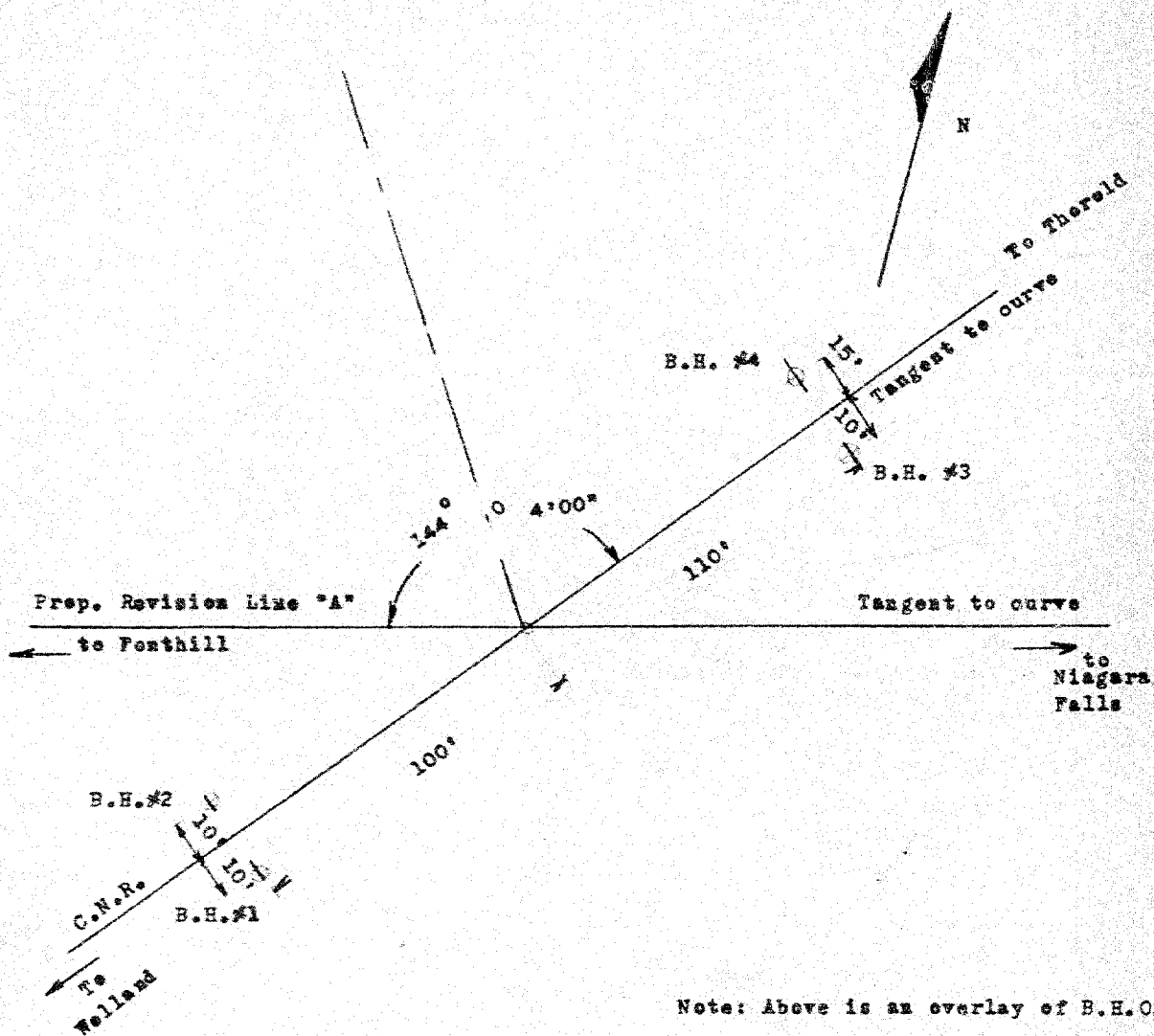
Foundation Office.

Gen. Files. ✓

Order No.

Enclosure No. 1.

Prep. By



Note: Above is an overlay of B.H.O.  
Drawing No. E-3097-1

PLAN OF BOREHOLE LOCATIONS

## RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 1.

Project: C.N.R. Overhead

Location: Allanburg, Ont.

Hole location as shown on enclosure No. 1.

Hole Elevation and Datum:

Field Work Begun

Ended

Field Supervisor: A.H.

Driller: Chevrier

Prep: J.S.

Checked:

## LEGEND

Sampling Method

2" Dia. split tube

2" Shelby tube

Penetration Resistance

2" Split tube

2" Dia. Cone

Casing

Strength

2" Shelby

Unconfined compression (Qu)

Vane test (C) and sensitivity (S)

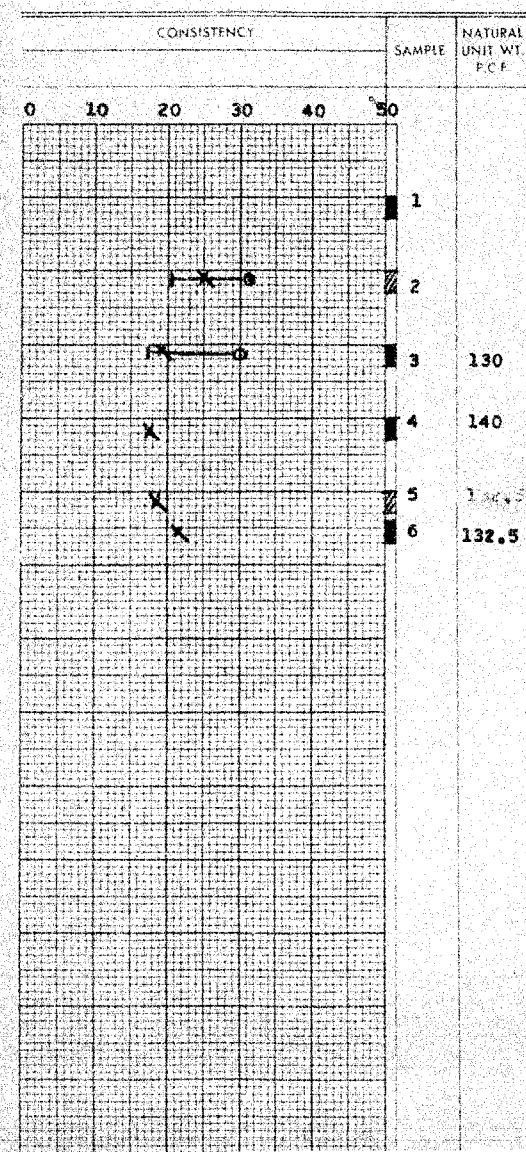
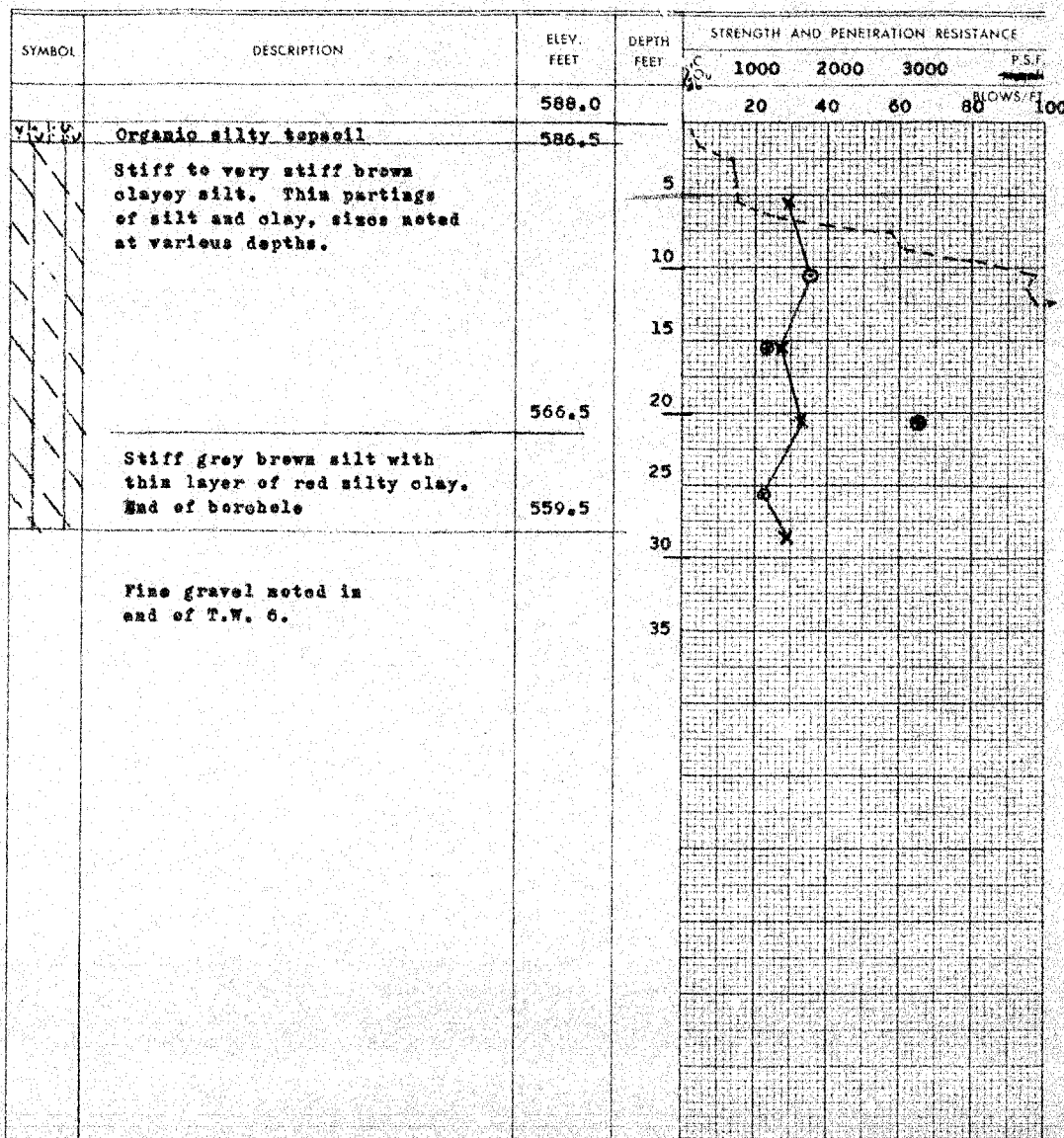
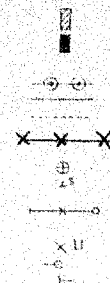
Consistency

Natural moisture and

Liquidity Index (LI)

Liquid limit

Plastic limit



## RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 2.

Project: C.N.R. Overhead

Location: Allanburg, Ont.

Hole location As shown on enclosure No. 1.

Hole Elevation and Datum:

Field Work Begun

Ended

Field Supervisor: A.H.

Driller: Chevrier

Prep.: J.S.

Checked:

## LEGEND

Sampling Method

2" Dia. split tube

2" Shelby tube

Penetration Resistance

2" Split tube

2" Dia. Cone

Casing

Strength

Unconfined compression (Qu)

Vane test (C) and sensitivity (S)

Consistency

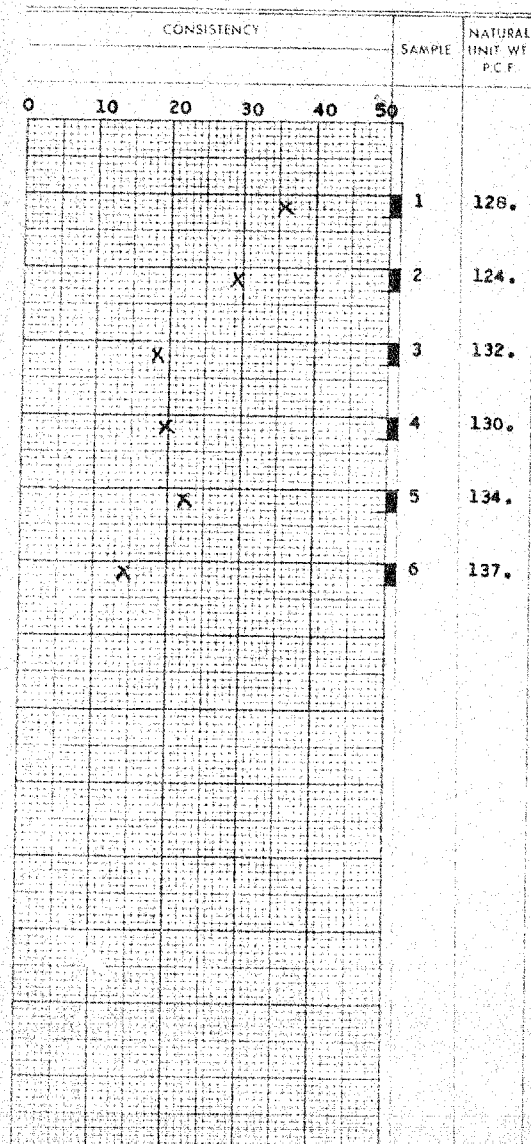
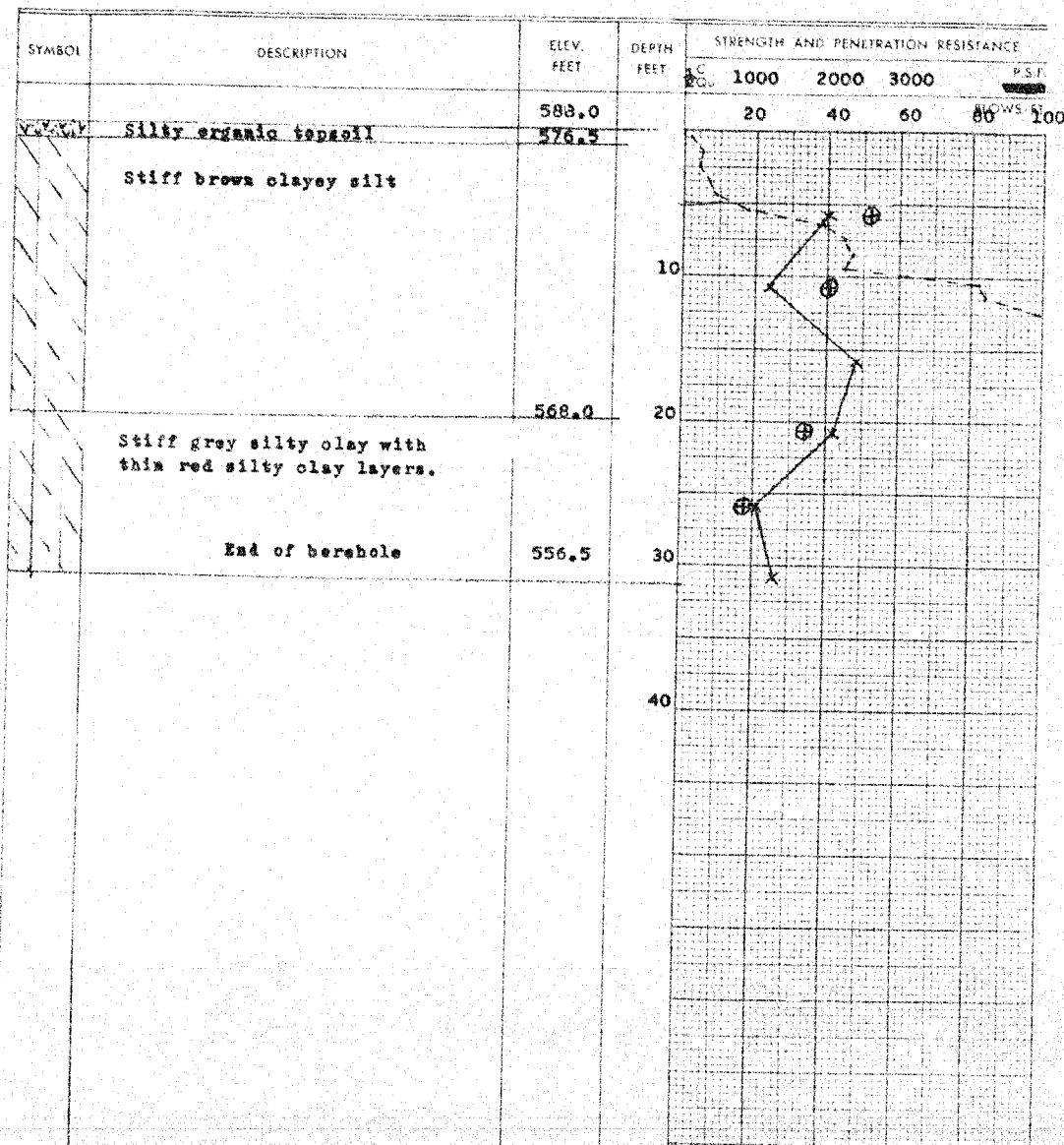
Natural moisture and

Liquid Limit (LL)

Liquid limit

Plastic limit

2" Shelby









## RACEY MacCALLUM AND ASSOCIATES LTD.

Foundation Engineering Division

Engineering Data Sheet for Borehole: 4.

Project: C.N.R. Overhead  
 Location: Allanburg, Ont.  
 Hole Location: As shown on enclosure No. 1.  
 Hole Elevation and Datum:  
 Field Work Begun

Ended

Field Supervisor: A.H.  
 Driller: Chevrier  
 Prep.: J.S.  
 Checked:

## LEGEND

Sampling Method

2" dia. split tube

2" Shelby tube

Penetration Resistance

2" Split tube

2" Dia. Core

Casing

2" Shelby

Strength

X Unconfined compression ( $Q_u$ )Vane test ( $C_v$ ) and sensitivity ( $S$ )

Consistency

Natural moisture and

Liquidity Index (LI)

Liquid limit

Plastic limit

